DYNAMIC COLLAPSE OF DUCTILE RC COLUMNS UNDER NEAR-FAULT EARTHQUAKE

Chiun-lin Wu¹, Yuan-Sen Yang¹, Shyh-Jiann Hwang², and Chin-Hsiung Loh²

ABSTRACT

Recently structural collapse has gained broader attention worldwide in view that collapse analysis is the critical component in fully realizing performance-based design concept especially for the near-fault and 2/50 hazard levels. Since 2004, NCREE has conducted a series of global collapse experiments on its 5m×5m earthquake simulator. This paper presents experimental results obtained from a ½-scale RC ductile portal frame. A near-fault record from the 1999 Chi-Chi earthquake was used as the input motion to yield structural hysteresis with post-peak behavior. Preliminary numerical simulation results are briefly discussed. It is noted that image-based measurement technique was employed to monitor large displacement induced in structural collapse. These test data sets serve as a great platform for developing reliable numerical analysis methods in subsequent studies.

Keywords: dynamic collapse, reinforced concrete, near-fault earthquake

INTRODUCTION

While considerable advances have been made in the use of analytical and/or numerical methods to evaluate seismic performance of civil structures, recently there is a clear trend that more RC collapse experiments are being conducted or planned worldwide to gain more knowledge on failure mechanism in view that the fundamental characteristics of structural collapse are not easily amenable to an analytical/numerical treatment at the present stage. According to solid mechanics theory, development of negative structural stiffness stems from P-Δ effects and/or material fractures. Although in the literature there are plenty of studies on how to incorporate P-Δ effects in response analysis, few are capable of successfully simulating fracture-induced collapse, especially in brittle shear failure. Accurate fracture-based numerical approach is technically sophisticated and might be economically unaffordable to most design firms. As such, collection of experimental data on structural collapse, in global and local manner, becomes very informative in developing computationally affordable macroscopic models. This study, by performing shake table tests on typical building columns designed according to past Taiwanese practice, expects to develop a reliable phenomenon-based hysteretic model with consideration of material post-peak behavior. This model, when combined with P-Δ effects, will be capable of predicting structural dynamic response under code-defined maximum considered earthquakes. In brief, collapse simulation in both experimental and analytical manners is getting much popular worldwide in the fields of earthquake engineering more or less in view of the following needs:

- To ensure energy dissipation capacity and collapse prevention: Previous seismic design code documents considered only 10%/50yrs. earthquakes (i.e. an average return period of some 500

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years) and a performance objective of life safety was set accordingly, which imposed a single control point over the structural nonlinear skeleton curve. This arrangement significantly lessened computational efforts and effectively simplified seismic design procedure, but could not ensure sufficient structural ductility during extreme earthquake events. To eliminate this drawback, performance-based design adds an extra control point at 2%/50yrs. earthquake level to ensure the ultimate performance objective of collapse prevention can be satisfied. Collapse analysis, therefore, should provide the designer and client with the guarantee that, during such extreme events, local collapse (or, component failure) may take place, but system collapse can be avoided with confidence.

- To offer house owners an option for custom-made buildings: Performance-based design framework enables a building structure performs according to owner-specified objectives under 10%/50yrs. and 2%/50yrs intensity levels. An enterpriser may specify a higher seismic standard for his headquarter to alleviate earthquake-induced loss due to interrupted operation.
- To reduce probability of casualties: A return period of 2500 years earthquake indicates an occurrence rate of 2% in 50 years. If collapse prevention can not be guaranteed at this hazard level, then it means a 2% probability in 50 years that residents could lose their lives in earthquake attacks.
- To distinguish unique characteristics of innovative structural systems from conventional systems: In current engineering practice, the seismic performance of a building is evaluated through its strength capacity and maximum interstory drift under design earthquakes. These years, innovative structural systems adopt smart material and advanced design concept to be equipped with cutting edge self-centering devices to reach the goal of seismic isolation and/or energy dissipation to minimize residual deformation. These structural systems may have comparable maximum drift as traditional buildings, but permanent deformation is significantly reduced. To classify substantial difference from conventional structural systems in seismic performance, a combined consideration of permanent deformation together with maximum drift may be required in the future evaluation framework. In this regard, consideration of collapse or post-yield behavior will help advance accurate prediction of these indices.
- Retrofit of older essential facilities such as schools, fire and police stations, etc.: During the September 21 (local time) 1999 Chi-Chi Taiwan earthquake, a large number of older buildings built before 1982 sustained severe damage and many others suffered from complete failure. Most older structures are prone to shear type of failure in a low ductility manner. A large portion of the elementary and high school buildings falls into this category. The Chi-Chi earthquake hit the central part of Taiwan at 1:47am, so these collapsed school buildings did not cause tremendous tragedy of students’ death. However, it becomes main concerns of the governments how to retrofit old school buildings that are identified in high risk of structural collapse in future earthquakes. To reach this goal, dynamic nonlinear behaviors of these low ductility columns must be first thoroughly understood.

DESIGN OF SHAKE TABLE TEST

Specimen Design

The test frame was designed to represent a real 4-story commercial-resident complex, which is quite popular in the central part of Taiwan. The two columns, interconnected by a strong beam, represent those at the soft 1st story (vertical irregularity) of the building, which is also typical in this type of open-front commercial complex. Beams and footings were designed strong enough to ensure plastic behavior occurs only in columns. The reinforcement details of specimen frames are shown in Fig. 1. A 1:2 scale model is selected following the design steps recommended by Tassios (1992) and bearing capacity limitation of NCREE’s shake table. Acceleration, stress, and geometry are the three independent scale factors selected for the 1:2 scale experiments. Gravitational acceleration, density, viscous damping ratio, modulus of elasticity, and Poisson’s ratio are the 4 physical quantities, scale factors of which remain unchanged. It, however, should be noted that the material related properties may not be the same as the prototype structure because of a more or less inevitable deviation in material properties (e.g., concrete compressive strength) due to the mix design of microconcrete material and production of D4 steel wires that were used for transverse reinforcements. D4 steel wires
were made through cold-rolling operation on bars of a slightly larger diameter, with a consequence of an increase in its yield strength and a significant ductility loss because appropriate heat treatment (annealing) was not performed. However, the 90°-hook ties opened up before yielding could occur on these D4 wires. Also, stress and strain rates will be slightly accentuated since a time compression factor of $\sqrt{1/2}$ is used. While it is unlikely to manufacture reinforced concrete scale specimens to be true replica models, additional masses were provided to better reproduce inertia effects, column’s stress state, and corresponding natural period of the target 4-story building, considering that in many cases the level of action-effects due to gravity forces (or, axial load) is of paramount importance for the ductility capacity of RC columns. Specific similitude requirements are imposed based on the following understandings:

1. Geometric similitude is considered essential due to constraints of the shake table specifications.
2. The stress-strain curves for model and prototype materials should be as much similar as possible to each other both in compression and tension.
3. Strains in the model and prototype at failure are at a similar level.

In addition to the above-mentioned similitude requirements, it is also hoped that the ratio of vertical load, overturning moment and lateral excitation of the model structure can be kept close, as much as possible, to its prototype counterpart, but in the meantime out-of-plane instability of the test frames is alleviated to a considerably lower level. Based on the above criteria, the time scale of input earthquake motions was scaled down accordingly. A total weight of 21 metric ton lead ballast ($0.1 f' A_g$) was added to reproduce axial loads of 1st story columns.

![Fig. 1. Configuration of shake table test on a 2-column RC frame: photgraphical view (left) and schematic diagram (right).](image)

![Fig. 2. Tensile strength tests of #3 steel rebar (left) and D4 wire (right).](image)
Table 1. Structural periods obtained from white noise excitation results.

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Weight (metric ton)</th>
<th>Structural Type</th>
<th>½-scale $T_1$ (sec)</th>
<th>½-scale $f_1$ (Hz)</th>
<th>Prototype 4-story $T_1$ (sec)</th>
<th>Lead Packets (metric ton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>8.72</td>
<td>Non-Ductile</td>
<td>0.366</td>
<td>2.73</td>
<td>0.518</td>
<td>21.35</td>
</tr>
<tr>
<td>3</td>
<td>9.42</td>
<td>Non-Ductile</td>
<td>0.427</td>
<td>2.344</td>
<td>0.604</td>
<td>21.35</td>
</tr>
<tr>
<td>4</td>
<td>9.775</td>
<td>Ductile</td>
<td>0.465</td>
<td>2.144</td>
<td>0.658</td>
<td>21.084</td>
</tr>
<tr>
<td>5</td>
<td>9.600</td>
<td>Ductile</td>
<td>0.394</td>
<td>2.54</td>
<td>0.557</td>
<td>21.092</td>
</tr>
</tbody>
</table>

Construction, and Material Strength Tests of Concrete and Steel Rebars

The frame specimen was constructed in an upright position and was moved into NCREE laboratory for storage 5 weeks after construction job was complete. The concrete mix was cast in two lifts, footings, and then columns and beams with a 1-week interval in between. After the construction was complete, wet curing was continued for another 2 weeks. Standard concrete cylinders (15cm diameter by 30cm high) were cast at the days of concrete pour, and then cured in the same condition as the frame specimen. Compressive strength tests of concrete cylinders were conducted at the same days of the tests. The average concrete compressive strength at test date ranges from 336 kgf/cm$^2$ to 355 kgf/cm$^2$. Tensile strength test results of #3 bars and D4 wire used as the longitudinal and transverse reinforcement of columns respectively are also shown in Fig. 2.

Experimental Setup

A photographic view of the experimental setup is shown in Fig. 1. A supporting steel frame system was provided inside the table to prevent unfavorable out-of-plane movement of the frame specimen. Another protective beam system was installed underneath the specimen with sitting bents outside the table to catch the specimen from hitting the shake table when global collapse occurred. The experimental setup aims for instrumented observation of global dynamic collapse of RC columns. To do so, load cells, accelerometers, Temposonics linear displacement transducers (LDTs), inclinometers, and strain gages were employed to collect experimental data of engineering interest, which are helpful in finding how negative stiffness takes place and how specimen is capable of remaining in stability when negative stiffness does occur. All these observations are very helpful in finding numerical solution methods related to dynamic stability problems to solicit the introduction of performance-based earthquake engineering.

Input Ground Motions

Because a 1:2 geometric scale factor was taken for the test specimen, input ground motions should then be adjusted using a time compression factor of $\sqrt{0.5}$ on the basis of keeping unchanged acceleration scale factor (= 1). In the test, the EW component of TCU082 accelerogram from the 1999 Chi-Chi Taiwan earthquake was applied as the input ground motion based on the following considerations:

1. Representative of main characteristics of near-fault earthquake motions in Taiwan. In particular, Station TCU082 is located in central Taiwan, and is close to the target building studied. Frequency content of the record consists of dynamic velocity pulses; however, its waveforms do not contain static fling step pulses, which are not in the consideration of this study.

2. In addition to long-period velocity pulses, the frequency content also consists of short to intermediate period motions such that a wide range of frequencies could be covered, and non-
stationary evolution of frequency contents as observed in ordinary earthquake motions could be put into consideration. As such, excitation force will be able to remain in the same intensity level even when columns sustain damage and structural period lengthens accordingly.

3. Finally, spectral values of selected ground motions have to meet the capacity limitation of the shake table.

The selected ground motions, after modulated with a trapezoidal frequency domain filter from 0.2Hz to 20Hz, were scaled to the PGA levels at which test specimens will experience global collapse.

Test Procedures and Results

During the past 2 years, a total of 4 portal frames were built at NCREE. This paper will focus on discussing collapse test results of Specimen 4. Before full intensity input earthquake motions were applied, low-level 50gal white noise excitation was first employed to determine structural periods of test frames; major structural characteristics of Specimen 4 and the other tested frames are all included in Table 1 for comparison. Full intensity motions were applied to specimens for observation of the flexural-shear-axial failure sequence. Progressive collapse snapshots of Specimen 4 are shown in Fig. 3. Major flexural cracking took place at north face and south face of the column at 8.54s and 8.92s, respectively. Serious concrete cover spalling occurred at 11.12s due to crushing compression. At the final stage of collapse (12s and after), lateral drift of the north column reached 20%, at which longitudinal rebar buckled seriously and shear cracking was formed due to fracture of column ties. In Specimens 2, 4, and 5 more flexural deformation was observed, while in Specimen 3 shear deformation made a major contribution in the column failure. Displacements of test frames were monitored using both Temposonics LDTs and consumer type miniDV camcorders that record 480i images, and results from Specimen 4 are plotted in Fig. 4 for visual comparison. Displacement histories were obtained from video films through image processing techniques. For this purpose, a small in-house computer code ImPro v.0.51 (see Fig. 5) was developed, which in general should feature the following components: (1) conversion of pixel into length unit (e.g., mm, in, etc.), (2) automatic tracking of target, (3) calibration of image distortion due to optical lenses, and stereo distortion resulted from geometric relationship between specimen and camcorder, (4) synchronization between video films, (5) synchronization of initial time and sampling rates between video films (1/30s) and shake table data acquisition system (0.005s). Since camcorders were zoomed in to record small local areas of the column hinges and were elevated at appropriate heights as the column hinges to minimize distortions as much as possible to a negligible level, the 3rd component abovementioned thereby has not yet implemented in ImPro v0.51 at the current stage. In addition, it is advised that fixed-focus filming is always preferred to minimize computational efforts in calibrating image distortion. For those who have sufficient budget and also need high accuracy, high-speed metric cameras will be a much better choice because they record non-interlaced images, and usually have a high resolution of at least 1000×1000 pixels, and will record high frequency signals such as velocity and acceleration. In this study, a resolution of 640×480 pixels was obtained in the avi-format files, and finally an accuracy of ±0.8mm (at column bottoms) ~ ±1.63mm (at column tops) was achieved depending on the area that the camcorder taped. From Fig. 4, numerical results from image processing look satisfactory, and observations show that camcorders yield longer displacement histories than Temposonics LDTs since Temposonics sensors reached their stroke limit before the test could be completed. Obtained hysteretic loop of Specimen 4 is shown in Fig. 6. Axial load, shear force, vertical displacement, vertical acceleration and lateral interstory drift of Specimen 4 north column are plotted in Fig. 7. The loss of gravity load carrying capacity of the column is not significant since flexural hinge made major contribution to structural collapse. A low axial load ratio of 10% was applied to the columns and due to dynamic effect of overturning moment, the axial forces of columns fluctuate about their initial values during the progress of system collapse. Same observation was obtained for vertical displacement and vertical acceleration at the top of the RC columns. The experimentally obtained hysteretic loop imposes important implications on engineering practice; especially the segment with negative slope will help in determining failure point of structural components and system. Fig. 6 plots Elwood-Moehle backbone curves with slight modifications from the authors against experimental curves. Included in the figure are base shear strength converted from nominal moment capacity following ACI procedures and shear strength calculated according to Sezen
(2000). The comparison shows that Moehle and his co-workers proposed a reasonable backbone curve prediction, which at the current stage is very helpful in promoting collapse consideration among engineering community. With more collapse experiments conducted worldwide in the near future, Elwood-Moehle empirical formula should be able to evolve into a new form with adequate accuracy.

![Fig. 3. Close up snapshots of collapse mechanism at the top of Specimen 4 north column when negative tangent stiffness took place.](image)

![Fig. 4. Specimen 4 roof drift histories obtained by Temposonics LDTs and image-based measurement.](image)

![Fig. 5. NCREE-developed digital image based measurement software, ImPro v0.51 (2006).](image)
Fig. 6. Experimentally obtained hysteretic loops of Specimen 4 (left) in comparison with Elwood-Moehle empirical curves (blue dashed lines).

Fig. 7. Relations between north column axial load, vertical displacement, vertical acceleration, shear force, horizontal displacement of Specimen 4.

**NUMERICAL SIMULATION**

This section presents two nonlinear dynamic numerical simulation analyses using OpenSees (McKenna and Fenves, 2000). The first one is a pre-test prediction, which was carried out before the Specimen 4 experimental collapse test to estimate the maximal displacement and shear force responses, and numerical model was based on the material tests completed before the experimental test. The second one is a post-test simulation with a tuned model, which was carried out after the collapse test. The post-test numerical model was tuned to approach the experimental measured shear forces and displacements. As shown in Fig. 8, the OpenSees-based numerical models in the simulations were mainly made of displacement-based beam-column elements with concrete patches and rebar fiber sections to simulate the flexural behavior of the RC columns. Shear deformation and failure behaviors...
were ignored to simplify the simulations. According to the calculated base shear strength backbone aforementioned, the flexural strength is smaller and should dominate the failure behaviors of the RC columns.

![Diagram](image1)

According to the calculated base shear strength backbone, the flexural strength is smaller and should dominate the failure behaviors of the RC columns.

**Pre-test Simulation**

The purpose of the pre-test numerical simulation aims to approximately estimate responses of experimental collapse test of Specimen 4, including maximal shear forces, displacements and the direction of collapse. Pre-test simulations were also used for selection of the input ground motion that is most potential in causing structural collapse. Considering uncertainty of material properties and construction quality, three numerical models with different strengths of the RC portal frame were built and used for the pre-test simulation (i.e., a strong model, an average model and a weak model). Six ground motions were separately employed as the input ground motions of Specimen 4. In this paper, the average model with the Chi-Chi earthquake ground motion TCU082ew achieved in a previous NCREE collapse test was discussed (Wu et al., 2004; Yang et al., 2004).

The comparison of the pre-test simulation and the experimental result is shown in Fig. 9. The maximal shear forces of the pre-test simulation and experimental result roughly matched to each other, but the displacement, base shear history and the direction of collapse did not. Although the difference between these two ground motions (see Fig. 10) may be a factor leading to discrepancy in structural response histories, it does not make the major contribution. Small responses at the first several seconds of the tests also show that the natural frequencies of the pre-test model and Specimen 4 are very different, indicating that they are different in initial stiffness values if it is assumed that the mass values of the two systems have very little discrepancy.

![Comparison between pre-test simulation (green lines) and experimental results (dotted blue lines).](image2)
Post-test Simulation

The post-test numerical model presented here was tuned for the basic consistency between the numerical model and Specimen 4. The tuning process focuses on the consistency of the first several seconds, which the system is still in the roughly linear stage. The authors believe that consistency at the linear stage is the foundation for further studies on nonlinear behaviors and failure modes. However, it should be noted that the consistency does not prove that the tuned parameters were more correct. A trial-and-error tuned model should be carefully used and can not be considered as a correct model, even if it matches the expected result.

The tuning parameters include steel and concrete material properties, and distances between longitudinal rebar. The initial Young’s modules of steel and concrete, and the rebar distances of the post-test numerical model were reduced for the consistency of initial natural frequency. The strengths of steel and concrete were also increased for the consistency of the maximal base shear force. Fig. 11 shows the comparison of tuned post-test simulation and the experimental results after a few trial-and-error tuning processes.

In this work, it is not difficult to tune the model for the response consistency before flexural cracks of columns took place at 8.92s. The authors could not find a parameter leading to the consistency between numerical analysis and experimental responses after flexural cracks took place at both columns. Further studies using more sophisticated concrete and steel material models and refined numerical models to account for concrete-steel bond slip and shear cracks at section or element levels will be considered to enhance the reliability of RC collapse numerical simulation.
CONCLUSIONS

Global collapse and dynamic structural post-peak behaviors were presented in this study using ductile RC columns as the test specimen and near-fault record TCU082ew from the 1999 Chi-Chi earthquake as the input motion. It is observed that a combination of lateral drift and shear strength reduction may be promising in serving as performance indices for predicting initial system collapse. Test results obtained provide a great database for calibrating existing numerical simulation methods to account for post-peak behaviors. This type of shake table tests may be used as benchmark problems for developing advanced numerical simulation methods. In addition, image-based measurement technique was introduced to monitor large displacement of the specimen with success. A set of pre-test and post-test numerical simulations were carried out, indicating that further studies using different numerical models at the material level, section level or element levels should be considered to enhance the reliability of RC collapse numerical simulation.

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REFERENCES


